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GEOSYNTHETIC REINFORCEMENT FOR EMBANKMENTS OVER DEGRADING DISCONTINUOUS PERMAFROST SUBJECTED TO PRESSTRESSING

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ABSTRACT: Failures of road embankments on discontinuous permafrost commonly occur during thawing of the foundation soil. As an approximation, weak zones created by thawing of discontinuous permafrost can be considered as 'voids' within the foundation. Geosynthetic reinforcements have been used to bridge these 'voids' and provide support to the embankment fill. This paper presents results of a numerical investigation on the effects of prestressing geosynthetics to enhance their reinforcement effect, and thus reduce deformations of embankments over discontinuous permafrost. The study used the commercially available computer program, FLAC. Numerical analysis illustrates that prestressing geosynthetic reinforcement can be effective in controlling deformations and reducing the possibility of collapse of road embankments on degrading discontinuous permafrost.

Keywords: Reinforced embankment, geosynthetic reinforcement, discontinuous permafrost, numerical modelling, pretensioning

INTRODUCTION

The foundation soil in many parts of Manitoba, Canada is glacio-lacustrine clay left by proglacial Lake Agassiz. As ice sheets melted northwards, runoff formed a large lake in what are now Manitoba, northwestern Ontario and northeastern Saskatchewan in Canada; and eastern North Dakota, and western Minnesota, both of which are US states. At its largest extent, Lake Agassiz was larger than all of the current Great Lakes combined. The present Lake Winnipeg is a remnant of this ancient lake (Coduto 1999). The Lake Agassiz deposit can be considered 'lacustrine lowland' in the terms used by Miura et al. (1994).

Many road embankments in Northern Manitoba experience lateral spreading that produces longitudinal cracking of the surface. Similar lateral spreading is also reported in Alaska (Kinney, 1993). The spreading may be caused by a number of factors that include thawing of permafrost, creep in the subgrade, and instability of the side slopes of the embankments. The rate of spreading can be considerable, and may, for example, require extensive patching several times per year to keep the road passable.

Roads in Northern Manitoba cross permafrost terrain that contains localized ice lenses/wedges and other ice masses of limited extent. That is, the permafrost is 'discontinuous'. Thermal characteristics of road surfaces are different from those of vegetated surrounding terrain. The different thermal regime can cause thermally stable discontinuous permafrost and ice to thaw beneath the road. This results in settlements that cause sharp and often irregular dips in the road surface that are dangerous to motorists and are expensive to repair. Figure 1 shows an example of such movements. The settlements and dips may form rapidly during summer and early fall, requiring reduced driving speeds and frequent repairs. The road may also undergo considerable lateral movements and failures during yearly spring thawing (Fig. 2).

Geosynthetics have been widely used to prevent collapse and control deformations of embankments on soft ground. They have also been used to reinforce embankments over discontinuous permafrost ground that includes 'complete voids' (Kinney and Connor 1987, 1990). The 'voids' were assumed to represent very weak zones in the ground caused by thawing of ice lenses/wedges (see Fig. 3). Kinney and Connor reported that properly selected geosynthetics could be used to span 'voids' of up to 3 m. This suggestion was based on measured surface displacements from test embankments. (Having established the usage, quotes '–' will no longer be used around subsequent references to 'voids'.)

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Note: Discussion on this paper is open until December 31, 2006

Fig. 1 Road sign warns drivers about sinkhole in Alaska (Stidger, 2001)

Fig. 2 Road embankment in Northern Manitoba showing signs of lateral movements and differential settlements (courtesy of Manitoba Transportation and Government Services)

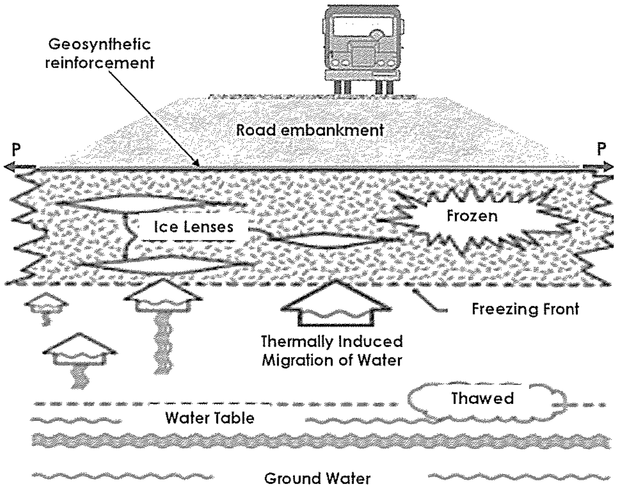


Fig. 3 Reinforced embankment on degrading discontinuous permafrost and mechanisms of ice lenses formation

An emerging technique for further improving the reinforcement of embankments over voids is the possibility of prestressing (pretensioning) the geotextile reinforcement (Koerner, 2000). Figure 3 shows how tension in the geosynthetic would go into action before significant deformations would occur in the composite soil-reinforcement system. It should be noted that without prestressing, tension in the reinforcement would be mobilized only when the composite system undergoes significant deformations.

A number of researchers have recognized the possibility of prestressing geosynthetics to improve its reinforcing effect. For example, Shukla and Chandra (1994) carried out analytical work and indicated that reduced settlements within the loaded area of a footing over soft clay ground can be achieved using low prestressing forces in the reinforcement. A laboratory-scale study was carried out by Leong et al. (2000) to investigate the performance of anchored and prestressed geotextiles in unpaved roads. The authors showed that prestressing the geotextiles before loading would induce tension in the geotextiles and hence mobilize membrane effects in the geotextiles without undergoing large deformations of the soil-geotextile composite system. Results of laboratory model tests and finite element analyses carried out by Xu et al. (2000) showed that prestressing the geosynthetics provides further reduction of shear stress levels in the soft ground, reduction of plastic zones, decreases in lateral displacements, and thus remarkable increases in embankment stability.

Construction engineers question how to apply prestressing forces to geosynthetic reinforcements in ways that are practical, maintainable, and cost effective. Chew et al. (2005) demonstrated a method for implementing field prestressing to geosynthetic reinforcement. Figure 4 shows the completion of prestressing work on a geosynthetic-reinforced embankment.

The first step is to anchor the geosynthetics in one side of the road using a trench drain. On the other side of the road, heaps of embankment fill or trench drain materials can be used as overburden to provide anchoring. Enough slackness of the geosynthetics is provided in the trench drain, which is tapered to eliminate sharp corners in the trench and prevent localized overstressing of geosynthetics. The amount of slackness in the geosynthetics in the drain is determined by estimating the amount of reinforcement strain required to achieve the desired prestressing force. To induce the prestressing force in the geosynthetics, an excavator can be used to press gravel placed on the geosynthetics in the tapered trench drain. To measure the strain across the reinforcement length, selected points can be marked on the geosynthetics prior to their

Fig. 4 Completion of prestressing work on geosynthetic reinforcement (after Chew et al. 2005)

installation. Measurements of relative displacements of the marked points are taken before and after prestressing to determine the amount of reinforcement strain, and thus the prestressing force due to prestressing. Chew et al. found that this procedure was able to achieve an average of 3.5% pretensioning strain across the geotextiles.

Research described in this paper used FLAC or Fast Lagrangian Analysis of Continua (Itasca Consulting Group, 2002). It involved numerical investigation of the effects of prestressing geosynthetics to enhance their reinforcement effect, and thus reduce deformations of embankments over discontinuous permafrost.

NUMERICAL ANALYSIS

Numerical methods for analyzing geosynthetic-reinforced embankments over voids assume that the soil and reinforcement rest initially on a firm foundation. With the development of a void under the reinforcement, the overlying soil deflects into the void. The deflection mobilizes two support mechanisms - (1) bending of the embankment soil and (2) stretching of the geosynthetic (Giroud et al. 1990). Bending of the embankment soil generates arching effects within the soil above the reinforcement and the load being transferred to the reinforcement over the void is less than the theoretical weight of the overlying soil. Stretching of the geosynthetic mobilizes part of the reinforcement strength and the material begins to act as a tension membrane supporting loads normal to its surface.

Analytical techniques used for design have until recently been based on a limit equilibrium method that uses combined arching and tension membrane theory. This approach uses two main steps in the analysis. First, the behaviour of the embankment soil is analyzed using classical arching theory to calculate the applied vertical pressure on the geosynthetics. Second, the required horizontal geosynthetic tension is determined using tensioned-membrane theory. In this approach, the soil response (arching) was uncoupled from the geosynthetic response (tensioned membrane) to simplify the complex nature of soil-geosynthetic interaction. Uncoupling the two mechanisms in this way means that the strain in the soil required to generate soil arching is compatible with the strain needed to mobilize tension in the reinforcement.

Tensioned membrane theory is based on two assumptions. One assumes that strain in the portion of the geosynthetic overlying the void is uniformly distributed. The second assumes that strain in the portion of the geosynthetic outside the void is zero and, that therefore, this portion does not slide towards the void. Although these two assumptions simplify the analysis, no attempt has been made to verify their validity.

The British Code of Practice, BS 8006 (BSI 1994) does not consider soil arching. Instead, it assumes that the full weight of an assumed wedge that forms above the void is supported by the reinforcement. It further assumes that the load acting on the reinforcement is distributed along the horizontal span of the reinforcement as opposed to being along the deflected length. BS 8006 does not address compatibility issues between the reinforcement and the soil. The calculated reinforcement load is therefore an upper estimate, since soil arching is not considered.

Recently, numerical methods based on continuum mechanics have been used to analyze geosynthetic-reinforced embankments over voids. Poorooshasb (2002) used a numerical technique based on an integro-differential (ID) equation in conjunction with a soil constitutive model to examine the behaviour of a geotextile-reinforced gravel mat bridging a cavity such as one that may be created by a sinkhole. Villard et al. (2000) used the finite element method to gain a better understanding of results from full-scale tests of reinforced fill over localized sinkholes. A finite difference computer program, FLAC was used to analyze the behaviour of a reinforced fill over a void (Agaiby and Jones, 1995; Kempton et al., 1996). This finite difference model uses a dynamic relaxation algorithm that is well suited to ill-behaved systems associated with material and geometric non-linearity, large strains, or where physical instability is anticipated. Although FLAC aims at providing static solutions to problems, dynamic equations are included in the mathematical formulations. The procedure first invokes the equation of motion to derive new velocities and displacements for stresses and forces. The strain rates are obtained from the velocities, and then new stresses are derived from the strain rates.

CALIBRATION OF NUMERICAL ANALYSIS

The authors have used the current version of the finite difference-based software FLAC, version 4.00 (Itasca, 2002) in their study of highway fills on degrading permafrost. To gain confidence, work began by simulating the problem studied by Agaiby and Jones (1995). The simulation was done to demonstrate that numerical results in the present study are in general agreement with results obtained by Agaiby and Jones using an earlier version of FLAC.

Figure 5 shows a schematic of the problem studied by Agaiby and Jones. A fill with a thickness, H = 2.0 m was constructed over a rigid formation. The bottom has a fixed boundary, while both sides of the problem have roller boundaries. A void of variable width B was considered to develop suddenly in the lower layer after the fill had been constructed. The void was bridged by geosynthetic reinforcement. A nominal surcharge of 20 kN/m^2 was applied on the fill surface to simulate traffic loads. The problem was studied for infinitely long fill, that is, for a plane-strain condition.

Relevant soil properties used by Agaiby and Jones (1995) include: soil density, ρ = 1750 kg/m^3, bulk modulus, K = 33.33 MPa, shear modulus, G = 15.37 MPa, cohesion, c' = 0 kPa, and ϕ' = 34°. The reinforcement was modelled as a series of cable elements that have no flexural rigidity and can only resist tension. Table 1 summarizes the properties used in the modelling for the reinforcement. Additional work has been done using stiffer reinforcement than that done by Agaiby and Jones (1995) Reinforcement R2 in Table 1 has a yield strength twice that of reinforcement R1, and stiffness 28% higher than R1. Figure 6 depicts the rotation of the major principal stresses due to arching in the fill. This demonstrates the capability of the numerical modelling to represent the problem being investigated in terms of soil arching and soil-reinforcement interaction.

Figure 7 shows results for the variation of normalized tension in the reinforcement and maximum surface displacements with the width B of the void. For reinforcement R1, which is the same as that used by Agaiby and Jones (1995), the authors' results and the Agaiby-Jones results are the same. For clarity, only the authors' results are shown in the figure.

As mentioned earlier, additional sets of simulations were also performed. One (R2) had higher reinforcement stiffness (Table 1). The second (R1P) had the same reinforcement stiffness as in R1 but the reinforcement was now prestressed. Two parameters are of interest, namely the maximum surface displacement, y_s and the maximum mobilized tension of reinforcement expressed as a ratio of yield strength, T/T_y. Figure 7 shows calculated values of y_s and T/T_y plotted versus the width of the void for the R1, R2 and R1P simulations.

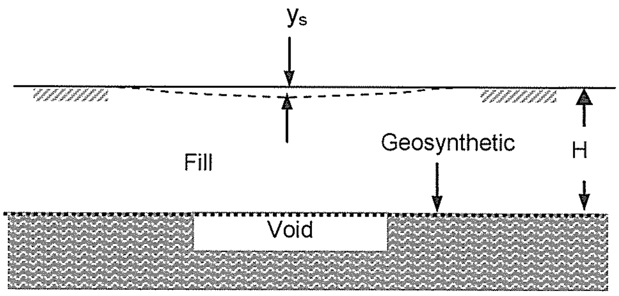


Fig. 5 Problem studied by Agaiby and Jones (1995)

Table 1 Geosynthetic reinforcement properties

Reinf.	Young's Modulus (GPa)	Area (10^{-3} m^2)	Yield Strength (kN/m)
R1*	2.35	1.7	400
R2†	3.00	1.7	800

* Same reinforcement used by Agaiby and Jones (1995)
† Arbitrary reinforcement

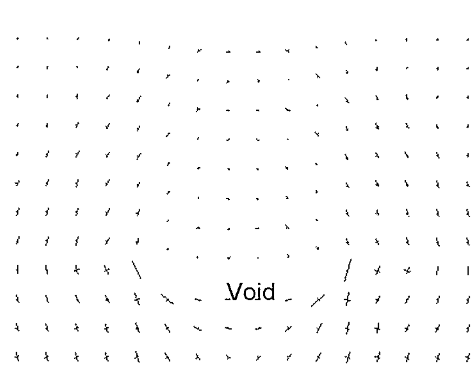
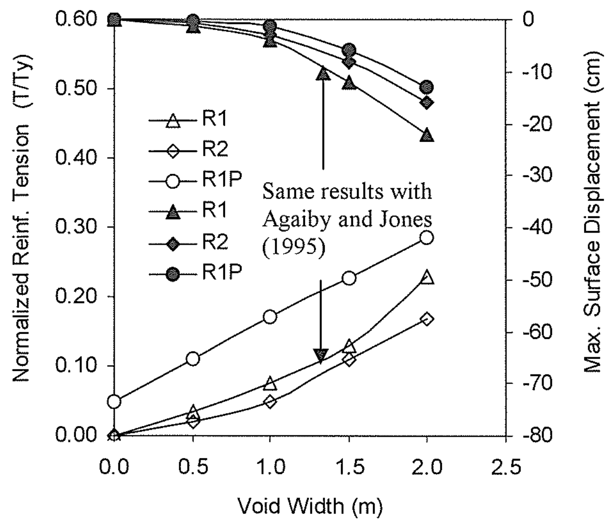


Fig. 6 Rotation of principal stresses due to arching in the fill overlying the void

Comparing the results for the R1 and R2 sets of calculations shows that the maximum surface settlements become smaller as the stiffness of the reinforcement is increased. Also, as the reinforcement becomes stronger, associated values of y_s and the normalised tension T/T_y become relatively smaller.

When initiating the research described in this paper, the authors proceeded on the basis that prestressing the reinforcement would enhance the reinforcement effect of geosynthetics and reduce deformations of the soil-reinforcement composite system. The prestressing



Solid symbols = Surface displacements;
Open symbols = T/Ty

Fig. 7 Maximum surface displacement and mobilized reinforcement tension for different widths of void

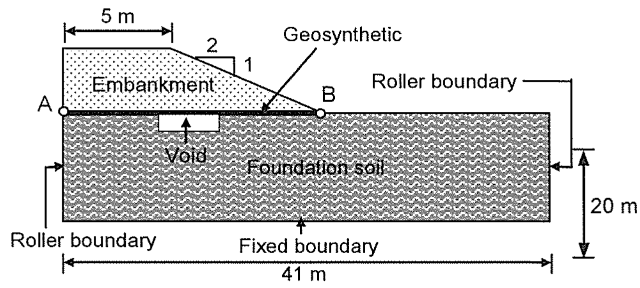


Fig. 8 Problem studied in this paper

Table 2 Soil properties used in this study

Soil type	ρ (kg/m^3)	G (MPa)	K (MPa)	c' (kPa)	ϕ' (°)
Found.	1640	0.30	1.67	50	20
Fill	1750	15.37	33.33	0	34

simulation (RP1) used a prestressing force of 20 kN/m in the reinforcement. (The initial tension represents a value of T/T_y = 0.05). Figure 7 shows that prestressing the weaker of the two reinforcements discussed earlier (R1) significantly increases the mobilized tension and reduces the maximum surface displacements. It is noted that geosynthetic reinforcements available in the market do not necessarily increase much in stiffness with increasing yield strength. This is particularly true at reinforcement strain levels of 5-10%, which are normally the working strain levels observed in most reinforced soil structures.

MODELLING OF REINFORCED EMBANKMENT OVER DISCONTINUOUS PERMAFROST

Problem Definition and Modelling Procedures

Figure 8 shows a schematic of a possible engineering solution to the real-life problem that led to this research project. The embankment fill is 3.0 m high with geosynthetic reinforcement laid at the base of an embankment. A void of 2.5 m in width was considered to develop in the foundation and was arbitrarily positioned as shown in the figure. A nominal surcharge of 20 kN/m^2 was considered to act on the top surface of the fill to simulate traffic loads.

The fill and the foundation soil materials were both represented by Mohr-Coulomb elements. Table 2 summarizes soil properties used in the modelling. These values represent the range of properties of Lake Agassiz clay found in most parts of Manitoba. The reinforcement layer was represented by cable elements with the properties given in Table 1. The interface between the reinforcement and the soil was modelled by grout material of negligible thickness with an interface angle, δ = 20°. In the absence of data from specific soil and reinforcement, the bond stiffness and bond strength of the grout were based on results of pullout tests reported by Alfaro et al. (1995). They were taken as k_b = 600 MN/m/m and s_b = 20 kN/m, respectively. The void is assumed to have no shear strength. That is, the pore water pressure is equal to the mean total stress. In fact, as ice in the foundation soil (or fill) melts, local high pore water pressures will dissipate towards regions of lower pressure. Round the void, there will be a gradient of decreasing water pressure, increasing effective stress, and increasing shear strength. In this early study of the effects of prestressing geosynthetic reinforcement, it has been considered sufficient to replace this complex relationship between ice, water and soil with a geometrically simple 'void' that averages out the effects of the melting.

The program FLAC uses stepped iterative procedures for achieving equilibrium. Equilibrium is considered to have been approached when the 'out-of-balance' forces are less than a user-defined value, which in this analysis was taken to be 0.1 N. The first step of the analysis was to equilibrate the foundation soil under its own gravitational force. This allowed the *in-situ* stresses to be calculated. A pretension force of 40 kN/m, similar to that of a pretensioned cable, was specified as the input parameter for the prestressed reinforcement. The inner end (Point A in Fig. 8) was assigned as a fixed end at the

Axial Force Maximum Value = 95 kN/m

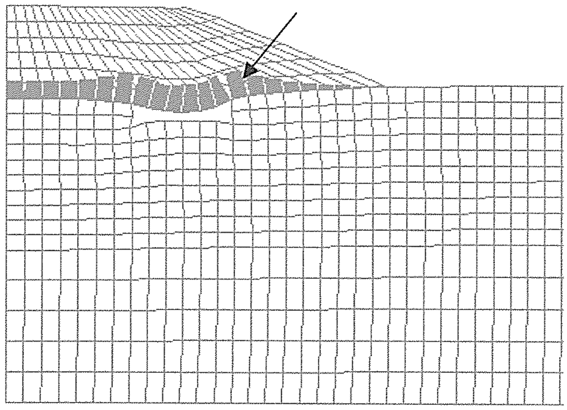


Fig. 9 True-scale deformed numerical grid and reinforcement load (Case 1)

node, with therefore, no relative movement with respect to the soil. The outer end of the geosynthetic reinforcement (Point B in Fig. 8) was assigned as a free end, that is, free to move relative to the confining soil. The embankment fill was then placed and the problem was stepped to equilibrium. The surcharge of 20 kN/m^2 was then applied at the top fill surface and again stepped to equilibrium. Then the void was formed and the problem was once more stepped to equilibrium.

Results and Analysis

The cross-section in Fig. 9 shows typical true-scale deformations for one of the reinforced embankments that were analysed. Superimposed on the figure are bar graphs representing the reinforcement load, T. The figure shows that the assumption by Giroud et al. (1990) of uniform tension in the reinforcement above the void is reasonably valid. However, the suggestion by Kinney and Connor (1990) that loads in the reinforcement in the embedded portion can be considered uniform requires caution.

Three cases were analysed. They correspond to three different levels of reinforcement, namely: Case 1 - using reinforcement R1 in Table 1, Case 2 - using reinforcement R2, and Case 3 - using reinforcement R1 plus a prestressing force of 40 kN/m (10% of the yield strength and twice the level of prestressing force used in RP1 Fig. 7). Although not shown here due to space limitations, the unreinforced embankment would undergo significant deformations leading to collapse if a void was to develop.

Figures 10, 11, and 12 show contours of vertical displacements for the three cases. The deformed grids in these figures show true-scale deformations. The ability of reinforcement R2 relative to R1 to reduce the maximum vertical displacements in the embankment fill is considered marginal (15% reduction). (Even though R2 is twice as strong as R1, it is only 28% stiffer, Table 1). However, prestressing the R1 reinforcement reduces the maximum vertical displacements significantly (40% reduction). Contours of corresponding lateral

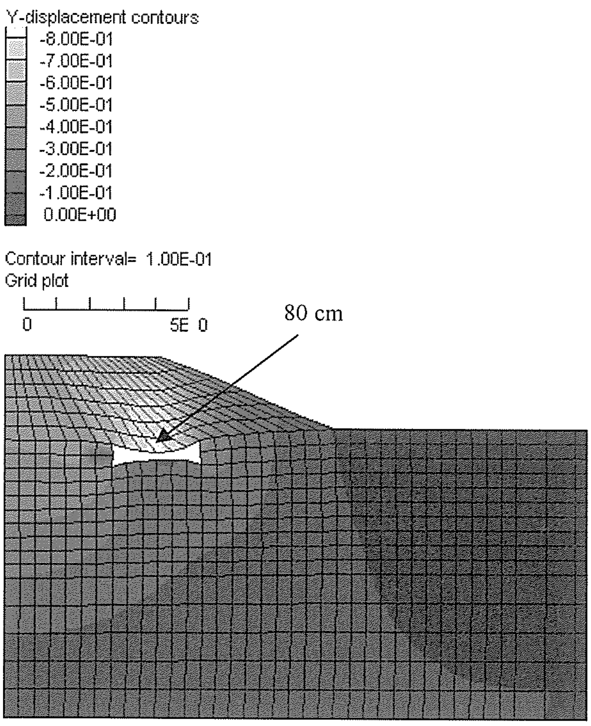


Fig. 10 Vertical displacement contours (Case 1)

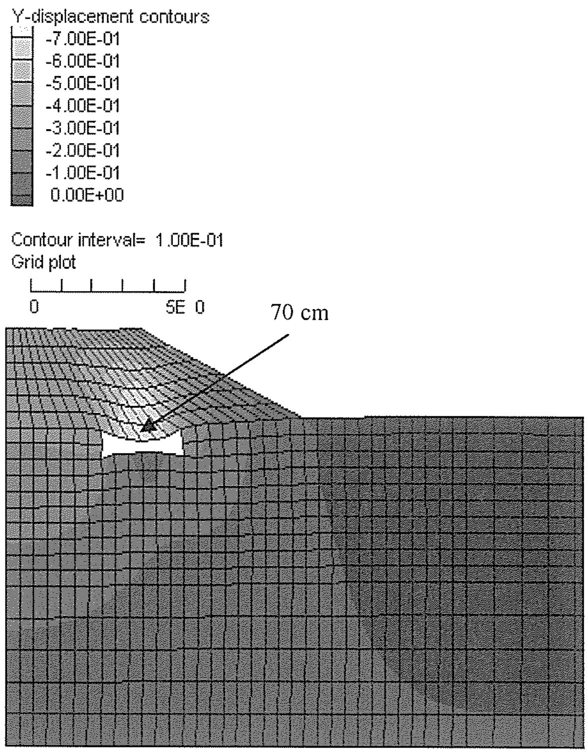


Fig. 11 Vertical displacement contours (Case 2)

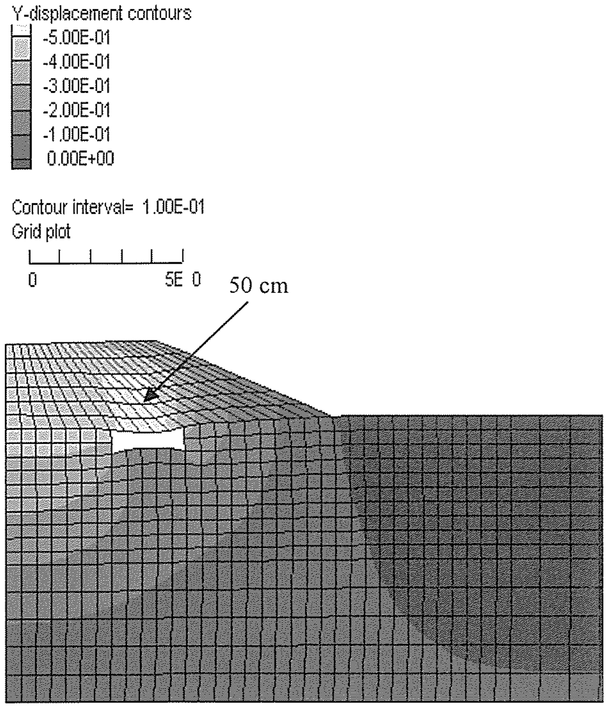


Fig. 12 Vertical displacement contours (Case 3)

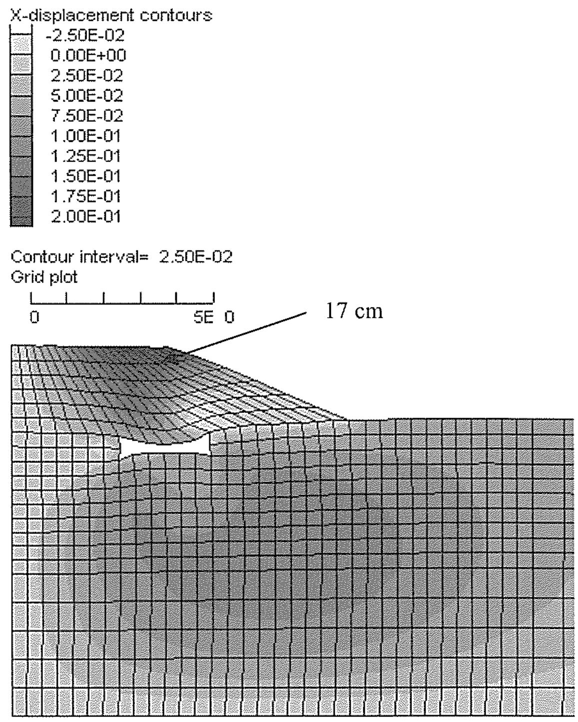


Fig. 13 Lateral displacement contours (Case 1)

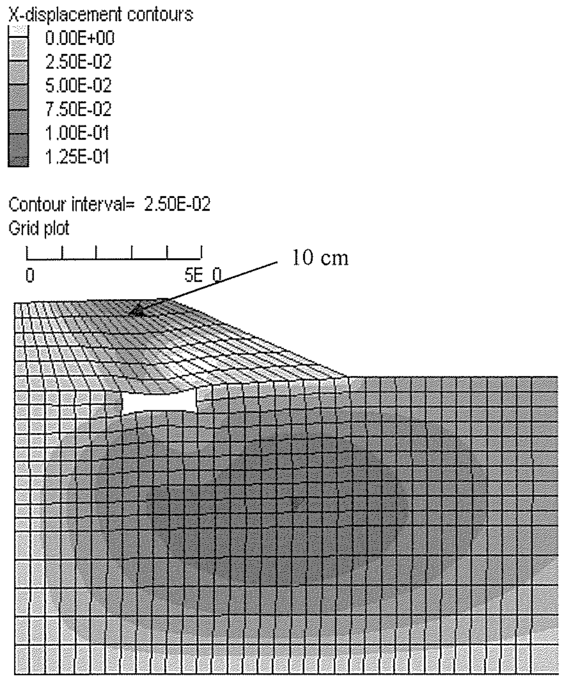


Fig. 14 Lateral displacement contours (Case 2)

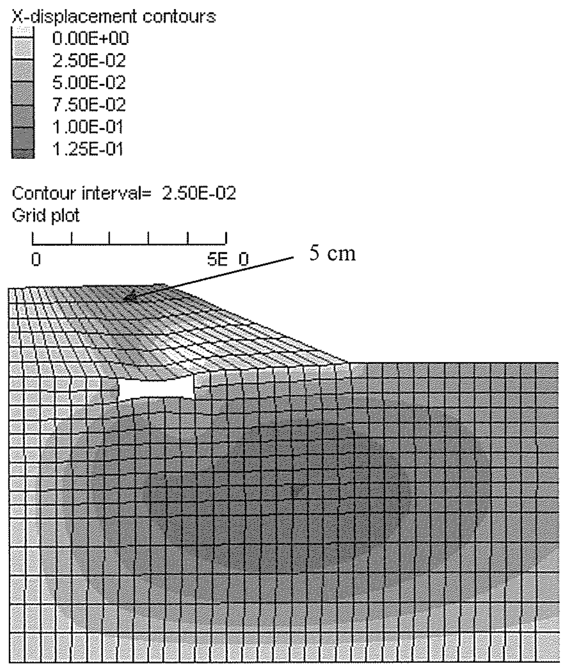


Fig. 15 Lateral displacement contours (Case 3)

displacements are shown in Figs. 13, 14, and 15. Here, the effects of prestressing the R1 reinforcement (RP1) are even larger, producing up to 70% reductions in lateral displacements in the embankment fill. Using unprestressed R2 reinforcement would improve lateral displacements by 40% relative to the performance of R1. Table 3 summarizes the modeling results.

Table 3 Summary of modelling results

Cases*	Max. δ_v (cm)	Reduction (%)	Max. δ_h (cm)	Reduction (%)
Case 1	80	-	17	-
Case 2	70	15	10	40
Case 3	50	40	5	70

δ_v = vertical displacement δ_h = lateral displacement

CONCLUDING REMARKS

Results have been presented from numerical analysis of the effects of prestressing geosynthetic reinforcements in embankments over permafrost soils that include rapidly-occurring voids caused by melting ice wedges. Qualitative results from this study illustrate that prestressing geosynthetic reinforcements can be effective in controlling deformations and reducing the possibility of collapse.

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